

## Chapter 3

# LOADS



East, Center, and West Bridges, Fairfield-Benton



Morse Bridge, Rumford

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### 3 LOADS

#### 3.1 General

The dead loads of the standard bridge components listed in Table 3-1 may be used for design computations or calculated separately at the option of the Structural Designer. The dead loads of the standard bridge components listed in Table 3-2 may be used for preliminary design only.

**Table 3-1 Component Loads**

<b>Bridge Component</b>	<b>Design Load</b>
Permanent Concrete Barrier Type IIIA	425 lb/ft
Permanent Concrete Barrier Type IIIB	600 lb/ft
2-Bar Steel Bridge Rail (without curb)	50 lb/ft
4-Bar Steel Bridge Rail - Traffic/Pedestrian (without sidewalk)	87 lb/ft
4-Bar Steel Bridge Rail - Traffic/Bicycle (without curb)	88 lb/ft
Texas Classic Bridge Rail - Traffic Rail (without curb)	300 lb/ft
Texas Classic Bridge Rail - Sidewalk Rail (without sidewalk)	371 lb/ft
Barrier Mounted Steel Bridge Railing - 1-Bar	9 lb/ft
Barrier Mounted Steel Bridge Railing - 2-Bar	19 lb/ft
3 inch bituminous wearing surface with membrane waterproofing	38 lb/ft <sup>2</sup>
2 inch un-reinforced concrete wearing surface	25 lb/ft <sup>2</sup>
Concrete Curb (20 inches wide with 3 inch bituminous wearing surface)	250 lb/ft
Concrete curb (20 inches wide with 2 inch concrete wearing surface)	220 lb/ft
Concrete curb with granite curb (24 inches wide with 3 inch bituminous wearing surface)	305 lb/ft
Concrete curb with granite curb (24 inches wide with 2 inch concrete wearing surface)	265 lb/ft

**Table 3-2 Component Loads for Preliminary Design Only**

<b>Bridge Component</b>	<b>Design Load</b>
Concrete sidewalk 5' wide (includes concrete under bridge rail)	1110 lb/ft
Concrete sidewalk 6' wide (includes concrete under bridge rail)	1290 lb/ft
Diaphragms for rolled steel beam	15 lb/ft per beam
Diaphragms for welded steel plate girder	20 lb/ft per beam

### **3.2 MaineDOT Live Load Policy (New and Rehabilitation)**

All new and replacement structures should be designed by AASHTO LRFD. The live load used is the code-specified live load for all limit states except for Strength I. The Live Load used for the Strength I limit state is the Maine Modified Live Load which consists of the standard HL-93 Live Load with a 25% increase in the Design Truck.

The magnitude of the design live load to be used in rehabilitating existing structures should be determined in each individual case, taking into account the inherent strength of the existing structure and the cost involved in providing additional load carrying capacity. In general, such structures should be strengthened to at least the code specified HL-93 live load for all limit states. A design capacity less than HL-93 must be approved by the Engineer of Design.

The optional deflection criteria (AASHTO LRFD Section 2.5) should be checked by the Structural Designer.

Load modifiers specified in AASHTO LRFD Section 1.3 relating to ductility and redundancy should generally be taken as 1.0. The use of non-ductile or non-redundant components is not allowed. The load modifier relating to operational importance should be taken as 1.0, unless otherwise indicated by the Engineer of Design.

Live loads determined by the AASHTO LRFD Specifications that are transferred to the substructure from the superstructure for geotechnical design will be unfactored. This unfactored live load will be used to perform a service load analysis according to the AASHTO Standard Specifications.

### **3.3 Thermal Effects**

The temperature range used to determine thermal forces and movements should be in conformance with the AASHTO LRFD "cold climate" temperature range.

### 3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

### 3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

### 3.6 Earth Loads

#### 3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

**Table 3-3 Material Classification**

Soil Type	Soil Description	Internal Angle of Friction of Soil, $\phi$	Soil Total Unit Weight (pcf)	Coeff. of Friction, $\tan \delta$ , Concrete to Soil	Interface Friction, Angle, Concrete to Soil $\delta$
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29° *	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

\* The value given for the internal angle of friction ( $\phi$ ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

### 3.6.2 Presence of Water

Retained earth should be drained and the development of hydrostatic water pressure eliminated by the use of a free-draining backfill such as crushed rock (less than 5 percent passing a No. 200 sieve), gravel drains, or other drainage systems. If retained earth is not allowed to drain, or if the groundwater levels differ on opposite sides of the wall, the effect of hydrostatic water pressure should be added to the earth pressure. Pore water pressures should be added to the effective horizontal stresses in determining total lateral earth pressure on the wall.

Walls along a stream or river should be designed for a minimum differential water pressure due to a 3 foot head of water in the backfill soil above the weepholes.

### 3.6.3 Earthquake

Where applicable, the effects of wall inertia and amplification of active earth pressure by earthquake should be considered. The Mononobe-Okabe method should be used to determine equivalent static pressures for seismic loads on walls and abutments as described in Section 3.7.3 Substructure. If the soils are saturated, liquefaction should be evaluated and addressed per Section 3.7.4.2 Liquefaction and Seismic Settlement.

### 3.6.4 Lateral Earth Pressure

The lateral earth pressure is linearly proportional to depth and is taken as:

$$\sigma = K \cdot \gamma_s \cdot z$$

where:

$\sigma$  = lateral earth pressure at a given depth,  $z$ .

$K$  = coefficient of lateral earth pressure, to be taken as:

$K_a$ , *active*, for walls that move or deflect sufficiently to reach the active conditions (refer to Figure 3-1)

$K_o$ , *at rest*, for walls that do not deflect or are restrained from movement

$K_p$ , *passive*, for walls that deflect or move sufficiently to reach a passive condition, including integral abutments.

$\gamma_s$  = soil unit weight (refer to Table 3-3)

$z$  = depth

The resultant lateral earth load due to the weight of the backfill should be assumed to act at a height of  $H/3$  above the base of the wall, where  $H$  is the total wall height, measured along a vertical plane extending from the ground surface above the back of the footing down to the bottom of the footing.

For walls with a total wall height,  $H$ , greater than or equal to 5 feet, the horizontal movement of the top of the wall due to structural deformation of the stem and rotation of the foundation is sufficient to develop active conditions.

At-rest earth pressures are usually limited to bridge abutments to which superstructures are fixed prior to backfilling (e.g. rigid frame bridges) or to cantilever walls where the heel is restrained and the base/stem connection prevents rotation of the stem.

### 3.6.5 Active Earth Pressure Coefficient

#### 3.6.5.1 Coulomb Theory

The Coulomb theory should be used for the design of the following yielding walls:

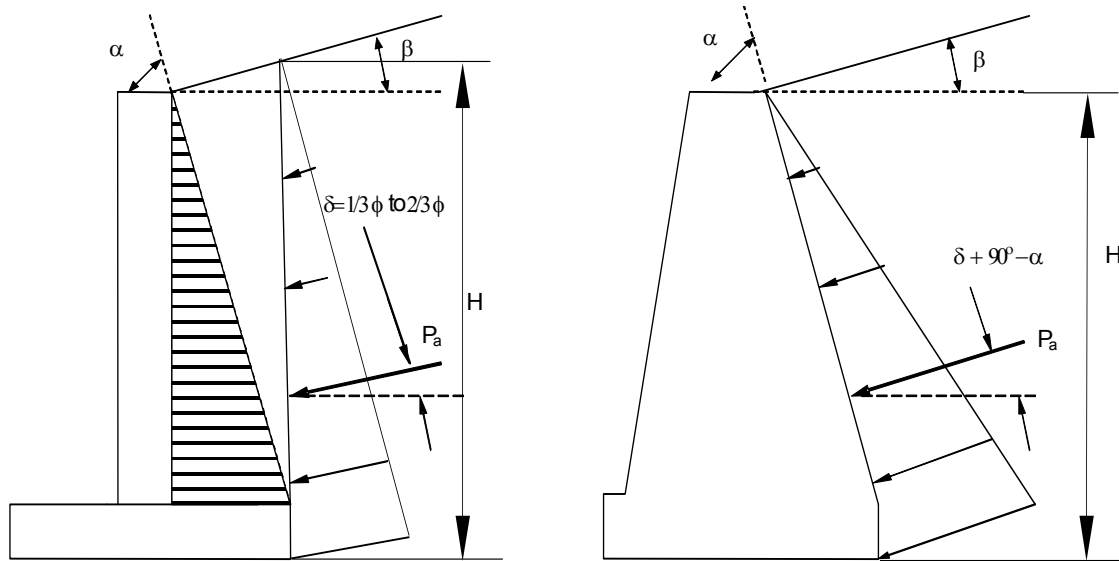
- Gravity shaped walls and abutments
- Semi gravity walls
- Prefabricated modular walls with steep back faces (20° or less measured from the vertical)
- Cantilever walls and abutments with short heels (refer to AASHTO LRFD Figure C3.11.5.3-1 (a) for the definition of short heel)

In all of these cases, interface friction ( $\delta$ ) develops along the back face of the wall. For horizontal or sloped backfill surfaces, the value of the coefficient of active lateral earth pressure (Coulomb),  $K_a$ , may be taken as:

$$K_a = \frac{\sin^2(\alpha + \phi)}{\sin^2 \alpha \cdot \sin(\alpha - \delta) \cdot \left( 1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\sin(\alpha - \delta) \cdot \sin(\beta + \alpha)}} \right)^2}$$

where:

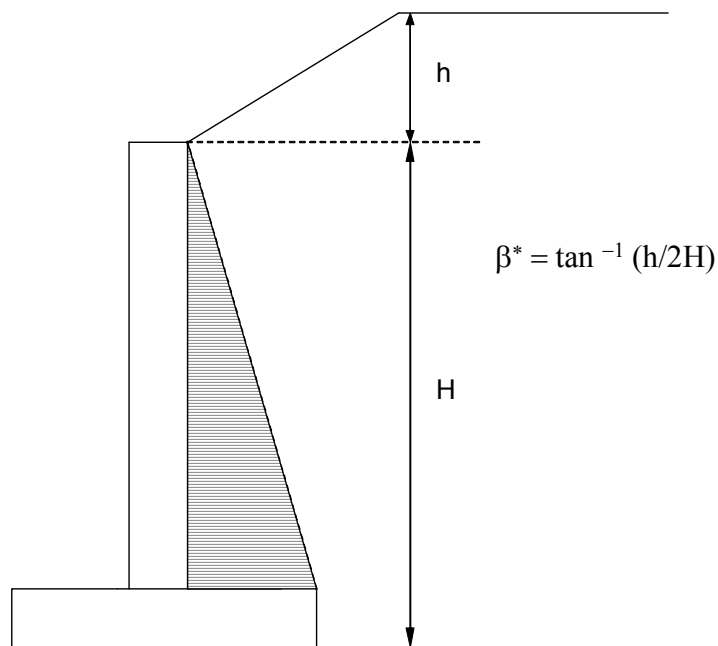
- $\alpha$  = angle (degrees) of backface of wall to the horizontal, as shown in Figure 3-1.
- $\phi$  = angle of internal soil friction (degrees), taken from Table 3-3.
- $\delta$  = friction angle (degrees) between fill and wall, taken from Table 3-3 for soil against concrete.
- $\beta$  = angle of backfill to the horizontal (degrees), as shown in Figure 3-1.



**Figure 3-1 Coulomb Theory**

The resultant earth pressure force,  $P_a$ , is oriented at an angle, either  $\delta$  or  $\delta + 90^\circ - \alpha$ , as shown in Figure 3-1. The resultant acts at a distance,  $H/3$ , from the base of the footing.

For situations with a broken backfill surface, the active earth pressure coefficient,  $K_a$ , may be determined using a  $\beta$  value adjusted per AASHTO LRFD Figure 3.11.5.8.1-3 or substituted with  $\beta^*$ , as shown in Figure 3-2.



**Figure 3-2 Calculating  $\beta$  with Broken Backfill Surface**

Rankine theory, as described in Section 3.6.5.2, may also be used for the design of yielding walls, for a simplified analysis (at the Structural Designer's option). The use of Rankine theory will result in a slightly more conservative design.

**3.6.5.2 Rankine Theory**

Rankine theory should be used for long-heeled cantilever walls. Refer to AASHTO LRFD Figure C3.11.5.3-1 (a) for the definition of a long heeled cantilever wall. For simplicity (at the Structural Designer's option), Rankine theory may also be used to compute lateral earth pressures on any yielding wall listed in 3.6.5.1 Coulomb Theory, although its use will result in a slightly more conservative design.

For these cases, interface friction between the wall backface and the backfill is not considered. Rankine earth pressure is applied to a plane extending vertically from the heel of the wall base, as shown in Figure 3-3.

For a horizontal backfill surface where  $\beta = 0^\circ$ , the value of the coefficient of active earth pressure (Rankine),  $K_a$ , may be taken as:

$$K_a = \tan^2 \left( 45^\circ - \frac{\phi}{2} \right)$$

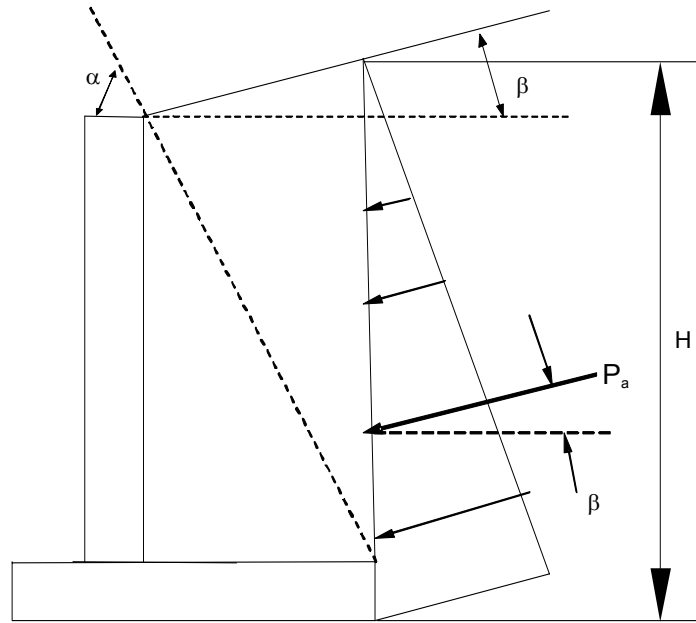
where:

$\phi$  = angle of internal soil friction (degrees), taken from Table 3-3.

$\beta$  = angle of backfill to the horizontal (degrees), as shown in Figure 3-3.

For a sloped backfill surface where  $\beta > 0^\circ$ , the coefficient of active earth pressure (Rankine),  $K_a$ , may be taken as:

$$K_a = \cos \beta \cdot \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}$$



**Figure 3-3 Rankine Theory**

The resultant earth pressure force,  $P_a$ , is oriented at an angle,  $\beta$ , as shown in Figure 3-3. The resultant acts at a distance,  $H/3$ , from the base of the footing.

For situations with a broken backfill surface, the active earth pressure coefficient,  $K_a$ , may be determined using a  $\beta$  value adjusted per AASHTO LRFD Figures 3.11.5.8 -1 through 3, or substituted with  $\beta^*$ , as shown in Figure 3-2.

### 3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

Values of the coefficient of passive lateral earth pressure,  $K_p$ , may be taken from Figures 3.11.5.4-1 and 2 in AASHTO LRFD or using Coulomb theory, as shown below:

$$K_p = \frac{\sin(\alpha - \phi)^2}{\sin \alpha^2 \cdot \sin(\alpha + \delta) \cdot \left( 1 - \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi + \beta)}{\sin(\alpha + \delta) \cdot \sin(\alpha + \beta)}} \right)^2}$$

where:

$\alpha$  = angle (degrees) of back of wall to the horizontal as shown in Figure 3-1.

$\phi$  = angle of internal soil friction (degrees), taken from Table 3-3.

- $\delta$  = friction angle between fill and wall (degrees), taken from Table 3-3 for soil against concrete.
- $\beta$  = angle of backfill to the horizontal (degrees), as shown in Figure 3-1.

The resultant passive earth pressure force,  $P_p$ , is oriented at an angle,  $\delta$ , to the normal drawn to the back face of the wall. The resultant passive earth load should be assumed to act at a distance of  $H/3$  measured from the bottom of the footing.

### 3.6.7 Lateral Earth Pressures for Unconventional Retaining Walls

#### 3.6.7.1 Mechanically Stabilized Earth Walls

For mechanically stabilized earth (MSE) walls, the resultant earth pressure,  $P_a$ , should be calculated using the active earth pressure coefficient,  $K_a$ , as described in Section 3.6.5.1 Coulomb Theory. For sloping and broken backfill surfaces, earth pressures should be calculated per AASHTO LRFD Figures 3.11.5.8 - 1 thru 3.

#### 3.6.7.2 Prefabricated Modular Walls

This category includes prefabricated concrete modular gravity (PCMG) walls, metal bin walls, and gabion walls. Where the back of the prefabricated modules form an irregular stepped surface, the earth pressure should be computed on a plane surface drawn from the upper back corner of the top module to the lower back heel of the bottom module using Rankine earth pressure theory. The magnitude and location of the resultant earth loads may be determined using the earth pressure distributions shown in AASHTO LRFD Figures 3.11.5.9 -1 and 2.

### 3.6.8 Surcharge Loads – Live Load Surcharge

A live load surcharge should be applied when traffic loads are located within a horizontal distance equal to one-half of the wall height,  $H$ , behind the back of the wall.  $H$  is defined as the total wall height, measured along a vertical plane extending from the bottom of the footing up to the ground surface at the back of the wall. The additional lateral earth pressure due to live load should be modeled by a surcharge load equal to that applied by a height of soil,  $H_{eq}$ , defined in Table 3-3. The surcharge will result in the application of an additional uniform, constant horizontal pressure on the back of the wall having a magnitude  $P_s$ , taken as:

$$P_s = H_{eq} \cdot \gamma_s \cdot K$$

where:

- $P_s$  = constant horizontal pressure due to live load surcharge  
 $\gamma_s$  = soil unit weight of soil, taken as 125 lb/ft<sup>3</sup>  
 $K$  = coefficient of lateral earth pressure, K, as defined in Section 3.6.4 Lateral Earth Pressure  
 $H_{eq}$  = equivalent height of soil for live load surcharge, determined from Table 3-4

The resultant horizontal earth pressure due to live load surcharge acts at the mid-height of the wall. The wall height is taken as the distance between the surface of the backfill and the bottom of the footing.

**Table 3-4 Equivalent Height of Soil for Calculating Live Load Surcharge**

Abutment or Wall Height (ft)	$H_{eq}$ (ft), Edge of Traffic is Normal to Wall or Abutment	$H_{eq}$ (ft), Edge of Traffic is Parallel to Wall and Located at Back of Wall	$H_{eq}$ (ft), Edge of Traffic is Parallel to Wall and Located 1 ft or More from Back of Wall
3	4	5	2
10	3	3.5	2
$\geq 20$	2	2	2

Note: Linear interpolation should be used for intermediate wall heights.

### 3.6.9 Passive Earth Pressure Loads

The resistance due to passive earth pressure in front of walls should be neglected unless the wall extends well below the depth of frost penetration, scour, or other types of potential disturbance, such as utility trench excavation in front of the wall. Neglecting this passive earth pressure is due to the consideration that the soil may be removed during future construction, which will eliminate its contribution to wall stability.

## 3.7 Seismic

### 3.7.1 General

The following criteria will be used to determine the scope of seismic analysis required.

#### 3.7.1.1 Seismicity of Site

According to AASHTO Standard Specifications Division I-A, Maine has a relatively low seismic risk. From Figure 3-4, it is noted that a portion of

southern, coastal Maine and of northern Maine are bounded by isoseismals of  $A = 0.10g$ . Bridges located in areas where the horizontal acceleration coefficient is less than or equal to 0.09 will be assigned to Seismic Performance Category (SPC) A. Bridges located in areas where  $0.09 < A < 0.19$  will be assigned to SPC B. AASHTO Standard Specifications Division I-A has not clearly defined the location of the 0.09 isoseismal for Maine, but Figure 3-4 provides this information. In this figure, an interpretation of the location of the 0.09 isoseismal was made through information provided by the Maine Geologic Survey. In general, SPC B will require a higher level of seismic performance analysis than SPC A.

#### 3.7.1.2 Geotechnical Characteristics of the Site

Soil conditions must be known to determine the seismic site coefficient for the bridge. In the AASHTO Standard Specifications Division I-A there are four soil profiles defined and a site coefficient is assigned to each profile. Additionally, potential hazards and seismic design requirements related to slope stability, liquefaction, fill settlement, and any increase in lateral earth pressures as a result of earthquake motion need to be identified. If required, the Geotechnical Designer will provide recommendations for site stabilization and design earth pressures.

#### 3.7.1.3 Functional Importance

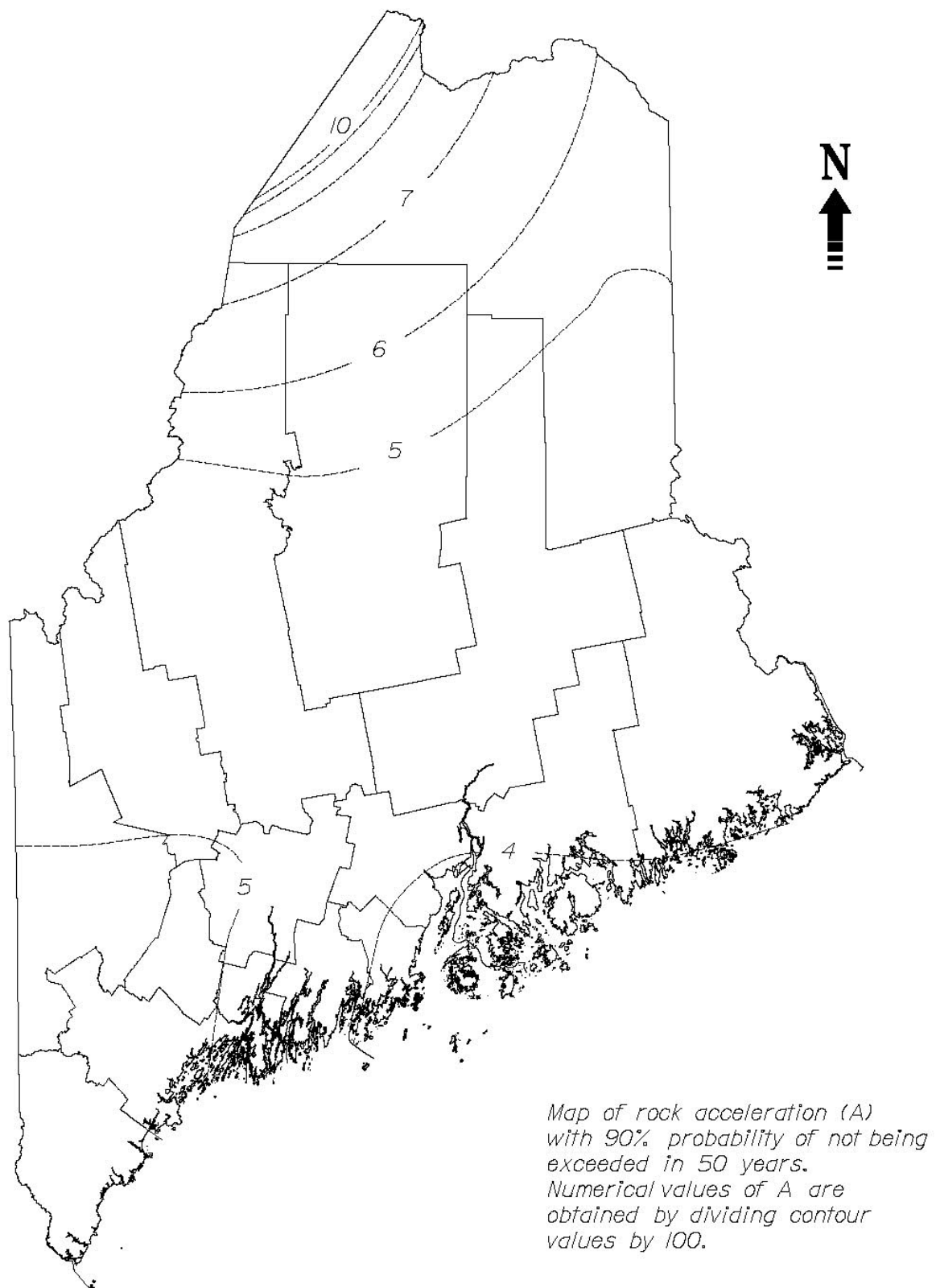
Bridges located on the NHS should be recognized as essential. Refer to AASHTO Standard Specifications Division I-A Section 3.3.

#### 3.7.1.4 Major or Minor Structures

Bridges are divided into two groups based on economics. Major bridges will be defined as those with bridge construction costs in excess of \$10 million. All other bridges will be considered minor bridges.

#### 3.7.1.5 Structure Type and Detail

Certain bridge types (e.g. multiple simple spans), or details (e.g. high rocker bearings) that are more vulnerable to earthquake damage should be avoided based on the probable severity of damage and the impact on the serviceability of the structure.



**Figure 3-4 Seismic Performance Categories for Maine**

Some other special conditions that are particularly sensitive to seismic forces are as follows:

- Single or individual column pier supports
- High, slender pier columns (where the slenderness ratio exceeds 60)
- Large skews (generally in excess of 45 degrees) with substandard support lengths
- Severe curvature where the subtended arc exceeds 75 degrees
- Unusual geometry causing portions of the structure to be significantly different in stiffness or that results in unusual support or framing details
- Hinges or seated connections in suspended superstructures
- Non load-path redundant superstructures

### 3.7.2 Seismic Analysis

Analysis is done based on two categories:

- o *SPC A Bridges:* AASHTO Standard Specifications Division I-A Section 4.5 indicates that for SPC A, no detailed analysis is required other than connection design and bearing seat length. For the MaineDOT Bridge Program, this will be amended such that all major and functionally important bridges (with two or more spans) in SPC A will be designed according to the requirements for SPC B with an acceleration coefficient of  $A = 0.09$ .
- o *SPC B Bridges:* For SPC B bridges with two or more spans, a detailed seismic analysis is required. AASHTO Standard Specifications Division I-A Section 4.2 indicates that a single mode spectral analysis is adequate for both "regular" and "irregular" bridges. A "regular" bridge is defined as one having no abrupt or unusual changes in mass, stiffness, or geometry along its length, and no large differences in these quantities (>25%) between adjacent supports. "Irregular" bridges are ones that do not satisfy the definition of "regular" bridges.

Structural Designers should make every attempt to avoid designing "irregular" bridges or bridges with special conditions and should adopt good structural form where possible. The basics of good structural form are:

- o *Simplicity* - it is best to ensure that the transfer of loads is by the shortest and simplest route possible.
- o *Symmetry* - seismic loads are inertial loads and act through the center of mass of each component while the resultant of the resisting force acts through the center of stiffness. In order for a bridge deck not to rotate, the eccentricity between the applied force and the resisting force should be zero. Symmetry requires that the various sources of lateral stiffness in a bridge (i.e., the piers and the abutments) be symmetrically located about the center of mass.
- o *Integrity* - This means that the various components of a bridge must remain connected together during an earthquake. Careful detailing is important. Generous girder seating lengths, conservative bearing details, confining steel in plastic zones, generous rebar anchorage lengths, shear keys, and other restraining devices are all examples of measures that will ensure a structure's integrity for seismic loads.

"Seismic Design and Retrofit Manual for Highway Bridges" (FHWA 1987) located in the Bridge Design Library, has several examples of acceptable structural form (refer to Chapter 5 Substructures). Structural Designers should refer to this and use it as a guide to design. Where it is impossible to avoid a structure that is "irregular" and located in SPC B, this manual recommends that a multi-modal method of analysis be done. This is because regular bridges are assumed to respond to earthquake loads in a single or fundamental mode of deformation. This is a reasonable assumption for regular, uniform structures, but may be in gross error for more complex structures. Irregular bridges can vibrate in other mode shapes besides the fundamental mode shape and still satisfy equilibrium. Irregular or unusual bridges are also likely to have higher modes, which will need to be considered.

The AASHTO Standard Specifications Division I-A provides guidelines on how to perform a single mode analysis. This method can be done manually using hand procedures or by computer methods. Usually the latter is preferred for all but the simplest bridges. General purpose space frame programs are capable of doing a single mode analysis through the use of the uniform load method.

### 3.7.3 Substructure

The recommended method of analysis of substructure units for seismic loads is described in Article 7.4.3 of AASHTO Standard Specifications Division I-A and the Specification Seismic Design Commentary. Additional guidance is provided in "Design Examples 1 through 7" (FHWA1997).

The recommended procedures include applying the Mononobe-Okabe Method of analysis for lateral earth overpressure, and accounting for the seismic

inertia forces of both the substructure self weight and the soil resting on the substructure footings. The earthquake overpressure force is equal to the total active earth pressure force (including seismic) as calculated by AASHTO Standard Specifications Division 1-A Equation C6-3, less the active (static) earth pressure. The earthquake overpressure force includes only the additional seismic pressure that occurs during an earthquake. The centroid of this additional force is assumed to act at a distance of  $0.6H$  above the top of the footing.

### *3.7.4 Embankments & Embankments Supporting Substructure Units*

#### *3.7.4.1 Seismic Slope Stability*

Seismic stability of slopes and embankments is evaluated using the Unified Methodology for Seismic Stability and Deformation Analysis. Refer to Chapter 7 of “Geotechnical Earthquake Engineering for Highways” (FHWA, 1997).

The Unified Methodology combines two accepted methods for seismic stability: the seismic coefficient-Factor of Safety (FOS) approach and the permanent seismic deformation approach. First, a seismic coefficient FOS analysis is performed. Then, if the seismic coefficient FOS analysis results in a factor of safety less than 1.0, a permanent seismic deformation analysis is performed.

A variety of computer programs can be used to perform both of these pseudo-static limit equilibrium analyses: PCSTABL4, PCSTABLE5, XSTABLE, and SLOPEW. Seismic loads depend on the bedrock acceleration at the site, and a seismic coefficient. Consult “Geotechnical Earthquake Engineering for Highways” (FHWA, 1997) for guidance on selection of a seismic coefficient. The Site Coefficient Factors (SCF) in the AASHTO Standard Specifications are for the structural and geotechnical analysis of walls and bridge foundations and are not applicable to slope stability and liquefaction analyses.

#### *3.7.4.2 Liquefaction and Seismic Settlement*

Liquefaction potential should be assessed employing the Simplified Procedure, originally developed by Seed and Idriss (1982) and progressively refined and summarized (FHWA, May 1997). For soil units in which the factor of safety against initial liquefaction is unsatisfactory, a liquefaction impact analysis must demonstrate that the site will still perform adequately if liquefaction occurs. Potential impacts of liquefaction include bearing capacity failure, loss of lateral support for piles, lateral squeezing, post-liquefaction-induced settlement, and downdrag. Liquefaction-induced

settlement of unsaturated sands must also be addressed as part of the post-liquefaction assessment (Tokimatsu and Seed, 1987).

If the seismic impact analyses yield unacceptable deformations, consideration may be given to performing a more sophisticated liquefaction potential assessment and to evaluation of liquefaction potential mitigation measures.

### **3.8 Non-Vehicular Bridges**

The design of prefabricated non-vehicular bridges should be in general accordance with the AASHTO "Guide Specification for Design of Pedestrian Bridges." Pedestrian bridges that are not prefabricated, long spans, or non-typical should be designed according to AASHTO LRFD Specifications. The optional deflection criteria and the use of load modifiers should be in accordance with Section 3.2 MaineDOT Live Load Policy (New and Rehabilitation).

The design live and dead loads of the bridge should be determined by considering several issues. For live loads, consider the width of the bridge, vertical clearance, use by emergency and maintenance vehicles, and use by snow grooming equipment. Dead loads should consider the type of rail, the use of a rub rail, security fencing, lighting, and any utilities (present or future). For further guidance on the applicability of dead and live loads, refer to Section 1.6 Non-Vehicular Bridges.

In general, a 10 foot wide non-vehicular bridge should be designed for the appropriate pedestrian load and an H5 (10,000 pound vehicle with 2,000 pound front axle and 8,000 pound rear axle) vehicular live load. The Structural Designer should be aware that some snowmobile grooming equipment can weigh close to 15,000 pounds with a distributed dead load of up to 400 pounds per square foot.

### **3.9 Ice Loads**

#### *3.9.1 General*

The following criteria are to be used when designing for ice loads. Static loading should be used when it is anticipated that ice may occur between two substructure units while having open water in an adjacent span. Static ice loads should be applied separately and not combined with dynamic ice loads. It is not necessary to design for uplift or ice jams except in very special circumstances.

### 3.9.2 Dynamic Loading

The north/south zone boundary passes through Rangeley, Guilford, Medway, and Houlton.

- o *Design Pressure:* 200 psi on pier nose @ Q1.1  
100 psi on pier nose @ Q50
- o *Coefficients:* Apply nose inclination, pier width, and ice thickness factors given in AASHTO LRFD
- o *Ice Thickness:* 2 feet in northern zone  
1'-6" in southern zone  
  
Add 6 inches when ice conditions are known to be severe. Rivers known to have severe ice conditions are the St. John, Allagash, Aroostook, Penobscot, Kennebec, and Androscoggin Rivers
- o *Transverse Force:* 30 percent of longitudinal force
- o *Point of Application:* Q50 or Q1.1 elevation

### 3.9.3 Static Loading

- o *Design Load:* 5 k/ft on pier side
- o *Point of Application:* Q1.1 elevation

## 3.10 Water Loads

Static water pressure should be determined in accordance with AASHTO LRFD Section 3.7.1. Consideration should be given to the following design water levels for all limits states:

- o *Design flood event* – Q50
- o *Normal high water* – Q1.1

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